Conceptual Design
Sunnyview Stream Restoration and Water Quality Improvements

Harford County, Maryland
McCormick Taylor Project No. 5067-01

July 2, 2007
Revised October 12, 2007
Final Report November 27, 2007

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1.0 INTRODUCTION

This report summarizes the stream stability investigation and concept improvement approach within an unnamed tributary to Bynum Run, adjacent to Sunnyview Road near Bel Air, Maryland. The objective of this investigation is to gain insight regarding the existing condition/level of disturbance of the proposed treatment site in order to prepare a restoration design for the project site. A thorough understanding of the current state of the stream and future changes that may occur will be used to guide stabilization efforts.

1.1 Project Description

This project for Harford County Department of Public Works entails the evaluation of the existing site conditions and establishment of a design approach to alleviate the embankment and channel erosion that is endangering property and contributing to excessive sediment loads to Bynum Run, the Bush River and the Chesapeake Bay. The project includes a stream restoration design and the identification of additional areas for possible water quality improvement within the watershed. The location of the project can be seen in *Figures 1 and 2*.

1.2 Stream Classification

This unnamed tributary is located in Maryland watershed 02-13-07 (Bush River Area) and is a tributary to Bynum Run. The stream is designated as a Use III waterway (natural trout waters) by COMAR 26.08.02. In-stream construction in Use III Waters is prohibited from October 1 to April 1, inclusive, during any year.

2.0 METHODOLOGIES

2.1 Literature and GIS Review

Prior to the field investigation, a thorough data review was conducted by referring to various documents and reference materials. GIS data, including topography, orthophotos, storm drains, property boundaries, roads, soils, and structures was provided by Harford County. Additional data included plans and computations for private stormwater management facilities in the watershed:

- Christ our King Presbyterian Church at Emmorton and Lexington Road, dated 1988
- Brook Hill Manor, in southernmost Town of Bel Air, dated 1997
- Gracelyn Community, near the stream crossing at E. Ring Factory Road, dated 1990

A copy of the *Sunnyview Pre-Construction Monitoring Report* prepared by KCI Technologies, Inc. in December 2006, revised April 2007, was also provided.

2.2 Stream Site Assessment

On May 29, 2007 McCormick Taylor, Inc. performed a preliminary site reconnaissance along an unnamed tributary to Bynum Run located adjacent to Sunnyview Road. In order to evaluate the existing condition and level of disturbance, the proposed treatment site was walked including the immediate upstream and downstream vicinity. Observations were made on general site conditions, including current and potential failure mechanisms, proposed work limits, general design reaches, and potential restoration treatments. In addition, field mapping and photos documenting the general site conditions and proposed work limits were generated. Photographic documentation of site conditions is included in Appendix B.

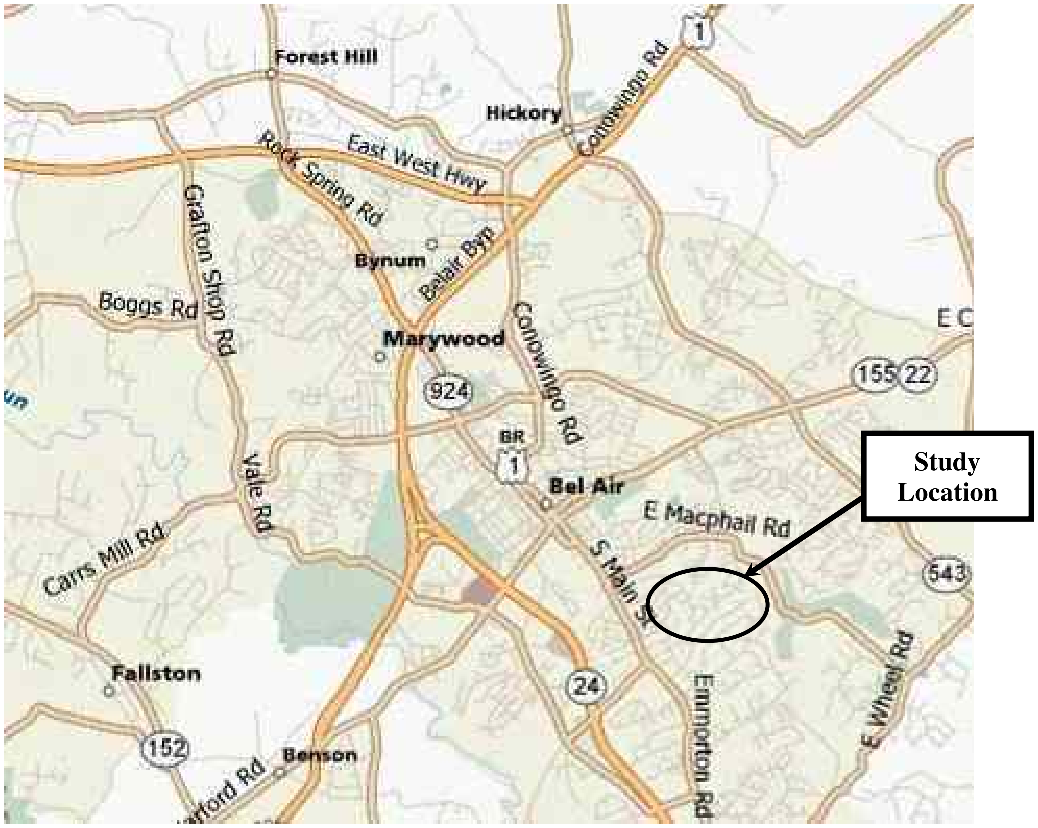


Figure 1: Vicinity Map

Figure 2: Location Map

STUDY LOCATION – Bel Air, MD, Harford County (not to scale)

2.3 Preliminary Stream Hydrology

The peak discharges for this project were developed using SCS TR-55 and the SCS TR-20 computer program, Version 2.04. Refer to *Appendix E* for the TR-55 input data and *Appendix F* for the TR-20 output. Results are noted in section 3.7. Methodology is detailed below.

Drainage Area
The drainage area for this study was delineated to the confluence of the unnamed tributary and Bynum Run and is approximately 577 acres (0.90 sq mi). This point is 70-80 feet downstream of the proposed channel improvements. Storage areas due to structures and individual storm drain systems which outfall into the stream were not modeled as part of this analysis. Based on the homogeneity of the land use and topography, and the relative size of the area, a single subwatershed was considered for this analysis. More detailed hydrology including routing of the watershed through the culvert at E. Ring Factory Road will be included in the Final Design.

Rainfall Data
The rainfall depths for the 24-hour duration storms were obtained from the Precipitation Frequency Data Server, maintained by the Hydrometeorological Design Studies Center (HDSC) of NOAA's National Weather Service (http://www.nws.noaa.gov/ohd/hdsc/). The new estimates, known as NOAA Atlas 14 Volume 2, replace those contained in Technical Paper No. 40.

Return Interval (yrs)	Rainfall Depth (in)
1	2.75
2	3.33
10	5.12
25	6.41
100	8.84

Table 1: Rainfall Data

Runoff Curve Number
Runoff curve number (RCN) was calculated for the 577 acre drainage area in accordance with SCS TR-55 (*Ref. 7*) using the hydrological parameters from land use, soil type, and ground slope. Curve numbers for good hydrologic soil conditions were applied to all land uses. The RCN value for the watershed is 72.7 for the existing conditions. Existing land use distributions and RCN values for the watershed are shown in *Appendix E.*

Time of Concentration
Time of concentration is the time required for runoff to travel from the hydraulically most distant part of the drainage area to a point of investigation in the watershed. TR-55 methodology was used to compute time of concentration from flow path hydraulics. A maximum length of 100 feet of overland flow was considered for this study. The land slope was calculated based the topographic survey. The roughness coefficient for channel flow was 0.040, based on field observation of its condition and the amount of vegetation on the banks. Cross section and length information was used in conjunction with the known slope to calculate an average flow velocity for the channel flow.

The time of concentration for the watershed is 0.876 hours. The time of concentration flow path is shown on the Drainage Area Map in *Appendix D*. Detailed time of concentration calculation data is provided in *Appendix E***.**

2.4 Best Management Practice (BMP) Site Assessment

During the desktop GIS analysis, and subsequent preliminary site reconnaissance on May 29, 2007, areas for possible stormwater quality management BMPs were identified. A second field visit was completed on September 17, 2007 to investigate additional privately owned existing stormwater management facilities (noted in section 2.1, above) for retrofitting and/or improvements. Seven possible sites were identified where runoff from significantly developed areas could be captured and treated. Two of the sites were existing failed SWM facilities in the Gracelyn development and the remaining five were located at existing storm drain or culvert outfalls. The sites were investigated and evaluated for several criteria including: the amount of impervious pavement draining to the area, the amount and slope of available land between the outfall and the unnamed tributary, conditions of the existing facility and possible permit issues (i.e. wetlands, Waters of the US) related to construction of a BMP. Additional discussion of this investigation can be found in section 4.1.8.

3.0 STREAM ANALYSIS

3.1 General Site Conditions

Overall, the unnamed tributary to Bynum Run located adjacent to Sunnyview is experiencing channel change and degradation associated with watershed development. Hydrologic response typically following watershed development includes increases in discharge magnitude and frequency and a decrease in lag time of storm hydrographs. Changes in hydrologic regime are usually followed by changes (i.e. increases) in channel width and depth to accommodate the increased magnitude and frequency of stream discharge.

Schumm (1969) provides qualitative expressions that describe the modes of channel change following an increase in water and/or sediment discharge. Independent variables of water discharge (Q_w) and sediment discharge (Q_s) are set approximately equal to dependent variables including width (w), depth (d), meander wavelength (L), slope (S), width-depth ratio (F), and sinuosity (P). Relationships presented by Schumm for increases in water discharge alone and increases in both water and sediment discharge are listed below, where the + and – signs indicate an increase and decrease, respectively:

- $Q_w^+ \approx \frac{w^+ d^+ L^+}{S^-}$
- $Q_w^+ Q_s^+ \approx \frac{w^+ L^+ F^+}{P^-} S^\pm d^\pm$

Based on the preliminary field investigation it is apparent that increases in both water and sediment discharge has occurred within the study site. It appears, however, that the increase in water discharge is proportionally higher than the increase in sediment discharge due to the increase in channel depth as a result of channel bed degradation.

While changes in hydrologic regime appear to be the main cause for channel change throughout the study reach, mechanisms of failure were identified that will aid in restoration treatment selection. Dominant mechanisms leading to bank erosion include toe scour and subsequent mass failure of upper bank material. Evidence of toe scour and mass failure mechanisms include sloughing of top-of-bank material along the bank toe and the existence of trees along the top-of-bank line that have slid partly down the bank as material below was eroded. It appears that bank erosion has progressed more rapidly where woody riparian vegetation is lacking, due to decreased root structure and roughness.

In addition to the mechanisms listed above, bank erosion appears to be associated with meander migration, including potential for meander chute cut-off development. Chute cut-off channels typically form as backwater conditions occur along the upstream limb of the bend then flows across the inside of the meander bend and reenters the channel on the downstream limb of the bend (Cramer et al. 2003). Most of the observed meander migration, and potential chute cut-off development, appears to be working toward relatively minor planform adjustments. The development of one potential chute cut-off channel at the upstream limits of the study area could lead to more substantial planform change; however, due to the presence of woody vegetation along the top-of-bank and riparian zone, it does not appear that full development of a chute cut-off channel is imminent.

3.2 Fluvial Geomorphic Data

Existing conditions fluvial geomorphic data was obtained from the *Sunnyview Pre-Construction Monitoring Report*. This information was utilized in lieu of collecting additional existing conditions longitudinal and cross-sectional data. Based on the existing conditions longitudinal survey, the overall average channel slope is 0.98 percent, including average channel slopes of 0.95 and 1.1 percent located upstream and downstream, respectively, of East Ring Factory Road. A graphical depiction of the existing conditions longitudinal profile is located in Appendix B of the *Sunnyview Pre-Construction Monitoring Report*.

Existing conditions cross-sectional data for the six permanent monitoring stations is reproduced in Table 2. This data represents bankfull channel characteristics; including width, mean depth, width-depth ratio, and cross-sectional area. As indicated in the *Sunnyview Pre-Construction Monitoring Report*, obvious bankfull indicators did not exist at each cross section due to extensive bank instability. Where obvious field indicators were lacking, best professional judgment was necessary to establish potential bankfull elevations. Graphical depictions of the surveyed cross sections are located in Appendix C of the *Sunnyview Pre-Construction Monitoring Report*.

Cross Section Location	Feature Description	Width (ft)	Mean Depth (ft)	Width-Depth Ratio	Cross-sectional Area (ft^2)	Estimated Discharge (ft^3/s)
Station 1+46	Pool	14.9	0.9	12.9	10.0	168.1
Station 7+78	Riffle	20.0	1.0	17.8	18.3	125.9
Station 12+23	Riffle	23.2	0.9	22.3	16.8	189.2
Station 12+70	Pool	15.0	1.2	11.9	17.5	165.3
Station 24+24	Riffle	20.9	0.7	24.7	12.7	163.1
Station 26+61	Pool	21.0	1.1	19.0	21.1	155.2

Table 2: Bankfull Dimensions based on Cross-sectional Survey Analysis

Additional fluvial geomorphic data reported in the *Sunnyview Pre-Construction Monitoring Report* includes stream bank and bed stability data and sediment transport data analysis. Bank and bed pins were installed to monitor erosion and sedimentation along the stream banks and bed. Data collection in addition to initial conditions data presented in the *Sunnyview Pre-Construction Monitoring Report* for the longitudinal profile, cross sections, and bank and bed stability is scheduled to occur following bankfull events. The report states that no bankfull events were sampled during the 2005 data collection period and that general channel inspections conducted during the sediment transport sampling events (April to June 2006) indicated that full survey of the banks was unnecessary. Thus, only initial conditions longitudinal, cross-sectional, and bank and bed stability data was presented in the *Pre-Construction Monitoring Report*.

Four sediment transport sampling data sets were presented in the *Sunnyview Pre-Construction Monitoring Report*. The discharges associated with the sediment transport sampling events range from 2.72 to 336 cfs. Based on the streamflow and sediment transport sampling data and the discussion within the *Sunnyview Pre-Construction Monitoring Report*, it is likely that the sediment transport data collected during the 2.72 and 336 cfs events may not represent accurate sediment transport conditions. Extremely low discharges, such as 2.72 cfs, make it difficult to generate accurate hydraulic properties including velocity and shear stress, which are typically used to predict sediment transport. In addition, the *Sunnyview Pre-Construction Monitoring Report* indicates that the large, out-of-bank flow event (i.e., 336 cfs) in combination with the sampling method may have resulted in erroneous results. The remaining two sediment transport samples representing 18.4 and 23.1 cfs appear to have provided good-quality data. The current and future sediment transport data will be used in more detail during subsequent design phases.

3.3 Proposed Work Limits

Proposed work limits were determined based on areas of significant channel destabilization, logical transitions between the natural system and restored reaches, and logical points of construction ingress and egress. The proposed work limits discussed below include the downstream- and upstream-most limits of proposed work. Additional transitions between the existing system and restored reaches within these overall work limits is discussed below in the design reaches section (3.4).

The channel was investigated in detail to approximately the western Bel Air Town limits, where a number of small tributaries confluence with the main channel. It is possible to find areas within the main channel in need of stabilization or restoration beyond this point. However, the forested buffer around the channel is wider upstream of this area with significantly greater distances between the channel and the surrounding structures and yards. For this reason, construction access would be difficult and disturbance impactive to forested areas, leaving the overall benefit more in question. Therefore, additional work beyond this point was not explored as part of this effort.

Downstream Proposed Work Limits

The downstream proposed work limits tie into the existing channel at the top of a riffle feature about 70 – 80 feet upstream of the Bynum Run confluence. It is important to tie into an existing stable feature in order to reduce the potential for channel changes occurring outside the work limits influencing the integrity of the restoration work. This riffle feature appears armored and vertically stable. The banks along the riffle feature, and extending to the Bynum Run confluence, have experienced erosion. This erosion is likely the result of historic down-cutting of the channel bed and channel widening due to increased storm runoff and base level lowering along Bynum Run. The top–of-bank line and riparian zone along this area is vegetation with trees and shrubs and appears stable. The bottom one-third of the channel banks is relatively bare and, thus, any further bank erosion within this area would likely result from toe erosion and subsequent mass-failure mechanisms. In addition, extending the proposed work limits to the Bynum Run confluence would have required impacts to a forested riparian zone and grading along Bynum Run to ensure a stable transition.

Upstream Proposed Work Limits

Identifying distinct work limits along the upstream end of the site was less clear. Similar to the downstream proposed work limits, the goal of the upstream proposed work limits was to tie into an existing stable feature (i.e., riffle) while balancing site access and impacts to existing forested resources. The intent of the upstream proposed work limits was to include, at a minimum, residential properties that were experiencing bank erosion with limited vegetative cover along the banks and riparian zone. These criteria resulted in locating the upstream proposed work limits

(Option A) about 80 – 90 feet upstream of the eastern Bel Air line. However, proposed work limits (Option B) located adjacent to Jackson Boulevard should be considered due to additional bank erosion and channel instabilities.

About 30 feet upstream of the Option A work limits, a six- to eight-foot high bank is experiencing erosion due to tight meander bend geometry. In general, the stream flows directly into the eroding bank and then flows out of the bend at nearly a right angle (90°). This type of flow pattern generates excessive turbulence and erosive forces along the eroding bank. It appears that more rapid bank erosion is partly mitigated through the generally cohesive nature of the bank toe and the woody vegetation along the top-of-bank. It is unclear at this time if channel planform reworking and/or bank stabilization would be necessary in this area.

Additional bank erosion due to tight meander bend geometry is located about 50 – 200 feet upstream of the Option A work limits. The riparian zone on both sides of the channel is forested; however, a chute cut-off channel appears to be in the early stages of development. The chute cut-off potential is located near the downstream one-third of the meander bend where toe scour is the dominant erosion mechanism promoting channel migration. Due to the relatively low bank height and presence of woody vegetation along the top of bank and riparian zone, it does not appear that development of a chute cut-off channel is imminent. Associated with the relatively low bank height and tight meander bend geometry, it appears that flow into and across the floodplain occurs frequently along the downstream one-third of the meander bend. While it does not appear that full development of a cut-off channel is imminent, treatment of this area should be considered as conditions will likely worsen. It appears that planform realignment would be necessary to stabilize this area.

Construction access to stabilize the bank erosion due to the tight meander bend geometry located between 50 and 200 feet upstream of the Option A work limits is recommended from Jackson Boulevard. A clearing in the riparian forest exists along the northwestern side of the channel, which would provide satisfactory construction access. This construction access would allow for stabilization of isolated areas of bank erosion between Options A and B. One area of bank erosion that would benefit from stabilization is located directly adjacent to Jackson Boulevard that, if left in its current state, would continue to erode and potentially undermine part of the road. Extending stream restoration activities to the Option B location is presented based on the bank erosion and tight meander bend geometry located between 50 and 200 feet upstream of Option A and construction access from Jackson Boulevard.

While the downstream proposed work limits appear clear, two Options (A and B) were proposed for consideration due to channel instabilities, site access, and potential impacts to adjacent forested resources.

3.4 Design Reaches

Preliminary design reaches were identified during the initial site reconnaissance, which will help direct additional assessments and subsequent design development. Identifying design reaches focused on segments of the study area that exhibit generally homogeneous characteristics and/or infrastructure. The site was initially segmented into two primary design reaches that extend upstream and downstream of East Ring Factory Road. As described earlier, conditions along the channel banks include erosion and alternating wooded and grass-lined riparian areas and, thus, did not indicate obvious design reach segmentation.

The channel bed sediment, and apparent slope, show more distinct changes and homogeneous reaches. Two distinct surface sediment distributions, likely due to changes in slope and hydraulic conditions, are apparent downstream of East Ring Factory Road. The first reach (Reach 1) extends from the downstream proposed work limits upstream to about 400 feet downstream of the East Ring Factory Road culvert. The bed surface sediment in this reach is dominated by coarse to very coarse gravel with inclusions of small to medium cobble. In addition, bedrock was observed along portions of the channel bed throughout this reach. Concrete/macadam aggregate extends across the channel bed in this area and marks the distinct channel change. The second reach (Reach 2) extends from this point upstream to East Ring Factory Road. The bed surface sediment in this reach is dominated by small gravel, sand and silt.

Similarly, two distinct bed surface sediment distributions are apparent upstream of East Ring Factory Road. The first upstream reach (Reach 3) extends about 200 feet upstream of East Ring Factory Road. The bed surface sediment in this reach is dominated by small to medium gravel and sand. The upstream limit of this reach is near the current sediment transport sampling station. The second upstream reach (Reach 4) extends from this point upstream to the upstream proposed work limits. The bed surface sediment in this reach is dominated by coarse gravel with minor inclusions of sand and small cobble.

3.5 Restoration Treatments

Identifying effective treatments for stream restoration and bank stabilization requires an understanding of dominant failure mechanisms. As described above, the dominant mechanisms leading to bank erosion include toe scour and subsequent mass failure of upper bank material and meander migration, including potential for meander chute cut-off development. It appears that bank erosion has progressed more rapidly where woody riparian vegetation is lacking due to decreased root structure and roughness. The restoration treatments identified to address the dominant failure mechanisms are listed in Table 3. Approximate locations of the proposed restoration treatments are depicted in Appendix C; typical sections and details of the proposed restoration treatments can also be found in Appendix C.

Failure Mechanism	Restoration Treatment
Bank Erosion -Toe Scour - Mass Failure of Upper Bank Material	***Bank Reshaping***—grade the bank to a stable slope, typically 1.5:1 to 2:1 ***Cross-Section Modification***—balance hydraulic and sediment transport continuity ***Stone Toe Protection***—where potential for excessive toe scour exists and along the outside banks along meander bends especially near infrastructure
Meander Migration -Chute Cut-off Potential -Threatening Infrastructure	***Slight Planform Modification***—to provide a stable radius of curvature
Lack of Riparian Vegetation	***Tree and Shrub Plantings***—to increase root structure and roughness where bank erosion is progressing more rapidly due to a lack of woody riparian vegetation

Table 3: Restoration Treatments Identified to Address Dominant Failure Mechanisms

Five general restoration treatments were identified to address the dominant failure mechanisms. Bank reshaping, cross section modification, and stone toe protection were identified to address bank erosion, including toe scour and mass failure of upper bank material. Bank reshaping would be utilized in areas of isolated bank erosion where cross-section modification is unnecessary. Cross-section modification would be utilized where an apparent imbalance of hydraulic and sediment transport continuity exists. Attaining channel bed stability is prerequisite to achieve bank stability. For both bank reshaping and cross section modification, stream banks would be graded to a stable slope, typically 1.5:1 to 2:1 (H:V), stabilized with natural fiber matting, and planted with live stakes. Stone toe protection would be used where the potential for excessive toe scour exists and the protection of infrastructure and/or residential property is necessary.

Slight planform modifications would be proposed where meander migration is threatening infrastructure or it appears that a chute cut-off channel may develop. This would occur primarily at existing meander bends where an unstable radius of curvature exists and is leading to bank erosion and channel instability. Slight channel shifts may also occur where intact riparian vegetation exists above a zone of bank erosion and attempts to keep the riparian vegetation intact are appropriate. In this case the channel would be shifted slightly away from the eroding bank so that the erosion can be stabilized using fill material as opposed to grading the eroding bank and requiring removal of the intact vegetation. Identifying locations where slight channel shifts should occur will be done during more-detailed design phases.

It generally appears that bank erosion has progressed more rapidly where woody riparian vegetation is lacking, due to decreased root structure and roughness. Riparian tree and shrub plantings would be proposed in order to increase soil stability through root structure and floodplain roughness. Consideration would be given to constraints resulting from infrastructure and residential property.

In addition to the restoration treatments discussed above, a cross vane is proposed about 60 feet downstream (East) of the East Ring Factory Road crossing. The purpose of this cross vane is to provide a smooth transition between the existing culvert and the restored reach, provide a slight backwater effect during low flow conditions to the existing riprap located about 20 feet upstream of the proposed cross vane, and to direct flow to the center of the channel to assist in stabilizing the immediate downstream channel realignment

The restoration treatments proposed to address the dominant failure mechanisms identified throughout the study reach will require more-detailed analyses during subsequent design phases. This is especially true for proposed cross-section modifications, which will require balance of hydraulic forces and sediment transport continuity. A discussion of the methods used to determine the proposed cross-section geometry during conceptual design phase is provided below.

3.6 Proposed Cross-Section Geometry

The intent of the proposed cross-section geometry analysis is to provide an approximate channel geometry that will provide stable conditions and will be used as a starting point for future analyses during subsequent design phases. The proposed cross-section geometry analysis relied on existing geomorphic data presented in the *Sunnyview Pre-Construction Monitoring Report* and the preliminary channel hydrology results provided below (section 3.7).

A key component to assigning cross-section geometry is identifying a design discharge, typically considered to be the bankfull discharge. A preliminary design discharge analysis was completed in order to identify an approximate range of potential design discharges to be used as an initial boundary condition for proposed channel cross-section modifications and to provide insight for the more-detailed design discharge analysis to be completed during subsequent design phases.

Bankfull discharge identified in the *Sunnyview Pre-Construction Monitoring Report* and recurrence interval discharges presented in the preliminary channel hydrology results were utilized for the preliminary design discharge analysis. Three riffles were surveyed during the pre-construction monitoring and bankfull stage was identified (see Table 2 above). The identified bankfull discharges include 126, 163, and 189 cfs. In addition, the *Pre-Construction Monitoring Report* identifies the bankfull discharge at the streamflow gage site as 193 cfs. It should be noted that the *Sunnyview Pre-*

Construction Monitoring Report indicates that obvious bankfull indicators did not exist at each cross section due to extensive bank instability. Where obvious field indicators were lacking, best professional judgment was necessary to establish potential bankfull elevations. Graphical depictions of the surveyed cross sections are located in Appendix C of the *Sunnyview Pre-Construction Monitoring Report*.

Bankfull discharge typically correlates to the 1- to 2-year recurrence interval discharge range. Results from the preliminary channel hydrology analysis indicate that the 1- and 2-year recurrence interval discharges are 200 and 324 cfs, respectively, using TR-20 methodology. The Fixed Regression Equation methodology indicates that the 1.25- and 2-year recurrence interval discharges are 163 and 286 cfs, respectively.

Based on the bankfull discharges identified in the *Sunnyview Pre-Construction Monitoring Report* and the preliminary channel hydrology results, it appears that the likely design discharge ranges from 163 to 200 cfs. The design discharge may be closer to 200 cfs as two bankfull discharges identified in the *Sunnyview Pre-Construction Monitoring Report* include 189 and 193 cfs. A design discharge range of 163 to 200 cfs will be used for the purposes of the conceptual stream restoration design phase. However, further field analysis is required to adequately identify a design discharge that will facilitate stable channel dimensions.

Utilizing a range of 163 to 200 cfs for the preliminary design discharge, the proposed cross-section geometry of a typical riffle and typical pool is provided in Table 4. The values reported in Table 4 were obtained using *The Reference Reach Spreadsheet v4.3L* (Mecklenburg 2006). The overall reach slope of 0.98 percent and a roughness (Manning's n) of 0.04 were used as boundary conditions. The pool dimensions are based on increasing the riffle width by 15 percent (generally ranges from 10 to 30 percent) and increasing the maximum riffle depth by one foot. The proposed riffle and pool geometry will be refined during subsequent and more-detailed design phases utilizing hydraulic and sediment transport analyses.

Please note that utilizing at-a-section hydraulics to estimate discharge in bends and pools produces results that are inaccurately high due to flow complexities that deviate from steady uniform flow conditions. It is anticipated that due to ineffective flow areas and secondary circulation patterns through the bends, the corresponding discharge for the indicated hydraulic geometry in the pool is closer to 200cfs than the 353 cfs that was calculated.

Feature Description	Width (ft)	Maximum Depth (ft)	Mean Depth (ft)	Width-Depth Ratio	Cross-sectional Area (ft^2)	Estimated Discharge (ft^3/s)
Riffle	22.0	2.5	1.7	12.7	38.0	196
Pool	26.0	3.5	2.2	11.7	58.0	353*

**See discussion above regarding accuracy of estimating pool discharge using at-a-section hydraulics*

Table 4: Bankfull Dimensions and Characteristics of Conceptual Cross-Section Geometry

3.7 Preliminary Channel Hydrology Results

As noted above, the 1, 2, 10, 25 and 100-year return interval discharges for the existing land use condition were computed using the TR-20 model. Additionally, the watershed was analyzed using the Fixed Region Regression Equations for the Piedmont Urban Region (Table 6, below) in accordance with the parameters outlined in the Hydrology Panel Recommendation Report (*ref. 5*). This analysis is automated through the *University of Maryland GISHydro 2000* computer program.

Return Interval Storm (yrs)	TR-20 (cfs)	Fixed Region Equation (cfs)
1 / 1.25	200	163
2	324	286
10	780	755
25	1156	1120
100	1881	1870

Table 5: Existing Hydrology Results

Piedmont (Urban) Fixed Region Regression Equation	Standard error (percent)	Equivalent years of record
$Q_{1.25} = 17.85\ DA^{0.652}\ (IA+1)^{0.635}$	41.7	3.3
$Q_{1.50} = 24.66\ DA^{0.648}\ (IA+1)^{0.631}$	36.9	3.8
$Q_{1.75} = 30.82\ DA^{0.643}\ (IA+1)^{0.611}$	35.6	4.1
$Q_{2} = 37.01\ DA^{0.635}\ (IA+1)^{0.588}$	35.1	4.5
$Q_{5} = 94.76\ DA^{0.624}\ (IA+1)^{0.499}$	28.5	13
$Q_{10} = 169.2\ DA^{0.622}\ (IA+1)^{0.435}$	26.2	24
$Q_{25} = 341.0\ DA^{0.619}\ (IA+1)^{0.349}$	26.0	38
$Q_{50} = 562.4\ DA^{0.619}\ (IA+1)^{0.284}$	27.7	44
$Q_{100} = 898.3\ DA^{0.619}\ (IA+1)^{0.222}$	30.7	45

Table 6: Piedmont Urban Regression Equations

Though the watershed parameters for use of this equation are near the outer limits of its applicability (the drainage area of 0.9 sq. mi. is near the lower data limit of 0.49 sq. mi., and the 35% impervious area near the upper limit of 42.8%) this method still provides a reasonable comparison for calibration of the TR-20 results. The results of the two methods are summarized above in Table 5. It is notable that the TR-20 results compare very favorably with the regression results, well within the standard error for this analysis. Based on this, the TR-20 hydrology will be used for the subsequent hydraulic analysis of the stream channel in the forthcoming design phase. Detailed TR-20 output data and regression calculations (which utilize the GISHydro land use data) are provided in *Appendix F*.

3.8 Stream Summary

As described above, the existing channel has experienced widening and deepening through bank erosion and bed degradation. The dominant failure mechanisms include toe scour and subsequent mass failure of upper bank material and meander migration, including potential for meander chute cut-off development. It also appears that bank erosion has progressed more rapidly where woody riparian vegetation is lacking due to the decreased root structure and roughness.

Proposed work limits were determined based on areas of significant channel destabilization, logical transitions between the natural system and restored reaches, and logical points of construction ingress and egress. The downstream proposed work limits are located at the top of a stable riffle feature about 70 – 80 feet upstream of the Bynum Run confluence. Identifying distinct work limits along the upstream end of the site was less clear. Two options (A and B) were proposed for consideration due to channel instabilities, site access, and potential impacts to adjacent forested resources. Option A is located about 80 – 90 feet upstream of the Bel Air line and Option B is located directly adjacent to Jackson Boulevard.

Four preliminary design reaches, two upstream and two downstream of East Ring Factory Road, were identified to help direct additional assessments and concept design development. Finally, likely restoration treatments were identified to address the observed dominant failure mechanisms, and include bank stabilization, cross-section modification, slight planform realignment, natural fiber matting, live stakes, stone toe protection, and riparian zone planting.

Conceptual cross-section geometry for typical riffle and pool features were proposed based on a preliminary design discharge range of 163 to 200 cfs. The preliminary design discharge is based on the existing geomorphic data presented in the *Sunnyview Pre-Construction Monitoring Report* and the preliminary channel hydrology results using the TR-20 and Fixed Regression Equation methods.

The TR-20 hydrologic analysis results developed for the downstream point of the channel compare favorably with regional regression analysis of the watershed, and will be the basis for subsequent detailed hydrologic and hydraulic design, including storage routing through the East Ring Factory Road crossing.

Permit requirements for this proposed work will include a Joint Permit Application for impacts to Waters of the US and potentially to wetlands in the surrounding impacted areas. A preliminary cost estimate for this work is provided under separate cover. Additional data required to proceed into the detailed design phase includes detailed topographic field survey, hydraulics modeling of the existing channel, floodplain and structures, bed sediment characterization, additional pre-construction monitoring data and sediment transport sampling data, additional geomorphic data on reference channel conditions, utility designation, and wetland delineation. An estimate of cost for detailed design will be developed once the proposed scope and limits of work have been determined with the appropriate community input.

4.0 WATER QUALITY IMPROVEMENTS

4.1 Possible BMP Sites

Seven sites were selected for additional field investigating to determine if the installation of a BMP or retrofitting of an existing BMP would be feasible. Sites were numbered 1 to 7. The locations of each site are shown in Figure 3.

Figure 3: Possible BMP Site Locations

4.1.1 Site 1 – 920 Fallen Stone Court

Site 1 is located immediately upstream of the unnamed tributary crossing under East Ring Factory Road. The 36" bituminous coated CMP storm drain outfall, refer to photo on left below, is located in the yard of 920 Fallen Stone Court. According to Gracelyn Section 1 As-Built Plans, dated April 1990, an infiltration device and gabion outlet was constructed at the culvert outfall. Due to the standing water observed during the site investigation, the infiltration device is not working properly and is clogged with sediment. The area draining to this site is approximately 18.9 acres and consists of residential areas. Approximately 47.7% (9.05 acres) of the drainage area is impervious.

The discharge from the storm drain system flows south along the East Ring Factory Road embankment towards the stream channel for approximately 70 feet. This area is well vegetated and appears stable, (photo below, right). Ponded water was observed at the outfall and in the pipe.

This site may be considered a good candidate for a BMP retrofit due to the large amount of impervious area draining to the site. However, the entire drainage area is fairly large and the outfall area is currently stable, providing some vegetative treatment of the discharge prior to entering the tributary, making it less in need of water quality improvements relative to some other sites.

4.1.2 Site 2 – 418 Sunnyview Road

Site 2 is located immediately upstream of the unnamed tributary crossing under East Ring Factory Road, across the tributary from Site 1 (outfall channels from Site 1 and Site 2 join the tributary at same river station on opposite banks - Site 1 on the north bank, Site 2 on the south bank). The 30" bituminous coated CMP storm drain outfall is located in the yard of 418 Sunnyview Road.

According to Gracelyn Section 1 As-Built Plans, dated April 1990, an infiltration device and gabion outlet was constructed at the culvert outfall. Due to the standing water observed during the site investigation, the infiltration device is not working properly and is clogged with sediment. The entire outfall is fenced; refer to photo above, left. The area draining to this site is approximately 19.7 acres of residential area. Approximately 26.1% (5.16 acres) of the drainage area is impervious.

The discharge from the storm drain system flows along the East Ring Factory Road embankment towards the stream channel for approximately 90 feet. The channel is lined with riprap and appears fairly stable; refer to photo on right above. One deep area of erosion was observed approximately half way between the pipe outfall and the tributary where the bed material had been washed out from under the geotextile fabric and riprap. The channel is fairly flat and the banks have woody vegetation. A large amount of ponded water was observed at the outfall and in the pipe. Debris is beginning to build up in the fence located in the outfall channel, causing a partial blockage of flow.

This site is considered a good candidate for a BMP retrofit due to the large amount of impervious area draining to the site and the erosion in the outfall channel which contributes to the downstream accumulation of sediment within the tributary. Water at the outfall could be considered Waters of the US; however, there are only storm drains, no natural channels, upstream of this point.

4.1.3 Site 3 – 208 East Ring Factory Road

Site 3 is located at 208 East Ring Factory Road. Two 18" RCP pipes and one 36" RCP pipe, refer to photo on left below, outfall convey storm drain runoff along the east side of the property for approximately 175 feet before discharging into the tributary. The outfall is stable and protected with riprap. Base flow in the pipe during the field investigation may indicate Waters of the US. The area draining to this site is approximately 8.9 acres and consists of residential areas. Approximately 15.7% (1.40 acres) of the drainage area is impervious.


The outfall channel appears stable but is fairly steep; refer to photo on right above. The property owners have constructed a small pedestrian bridge near the house to access the other side of the outfall channel.

Although this may have appeared to be a possible location for a BMP facility based on the criteria, the slope of the outfall channel and permitting concerns due to the possible Waters of the US, this is a less favorable candidate for BMP placement.

4.1.4 Site 4 – 113 & 115 Colony Place

Site 4 is located between 113 and 115 Colony Place. A 12" RCP storm drain system outfalls behind a fence into a concrete channel; refer to photo on left below. The concrete channel follows the slope of the adjacent ground before steeply dropping approximately 3-4' into the streambed; refer to photo on right below. However, the stream bank is lined with a mix of riprap and stone at this connection.

Based on the GIS information and the field verification, it appears the outfall pipe may have been extended. The concrete channel is approximately 20 feet. The area draining to this site is approximately 2.9 acres and consists of residential areas. Approximately 32.2% (0.94 acres) of the drainage area is impervious.

Due to the percentage of impervious area draining to this area and potential for increased outfall stability, we believe this site is a good candidate for a BMP facility or outfall improvement. Currently, any sediment or debris conveyed within the stormwater runoff is conveyed directly into the tributary and the tributary embankment is eroding at the steep drop. Removing the concrete channel and stabilizing the embankment at the confluence would reduce the sediment entering the tributary.

4.1.5 Site 5 – 130 Fairmont Drive

Site 5 is located in the backyard of 130 Fairmont Drive. This site was selected based on the GIS data; however the pipe outfall could not be located in the field. The area surrounding the tributary in this location was heavily vegetated in the vicinity of the storm drain outfall. The tributary appeared to flow through two culverts upstream of where the culvert for Site 5 was supposedly located; refer to photo on left below. The tributary appeared stable with bedrock; refer to photo on right below. It could not be determined if the GIS data was incorrect or if the existing pipe had been blocked.

The area draining to the site is approximately 24.5 acres and consists of residential areas. Approximately 27.6% (6.75 acres) of the drainage area is impervious. Further investigation, including additional property access permission through this property, is recommended. At this point, it can not be determined whether this site is a good candidate for a BMP facility.

4.1.6 Site 6 -102 & 104 East Ring Factory Road and 1215 & 1217 Vermont Road
Site 6 is located in the backyard of several properties along East Ring Factory Road and Vermont Road. Two separate storm drain systems outfall into a single outfall channel, which conveys flow towards the east approximately 150 feet to the tributary. A combination brick and concrete endwall has been constructed at the outfall; refer to photo on left below. Ponded water was observed at the outfall and in the adjacent channel, however both areas appear stable.

The primarily residential area draining to this site is approximately 44.0 acres. Approximately 22.8% (10.04 acres) of the drainage area is impervious. This site may be considered a good candidate for a BMP facility due to the large amount of impervious area draining to the site. The outfall channel is stable; however there is no removal of sediment prior to the confluence with the tributary. Refer to photo on right below. Environmental permits may be required for work within the channel since ponded water (possible Waters of US) is present.

4.1.7 Site 7 – 113 Sherwood Place and 1115 Emmorton Road
Site 7 is located in the backyard of 113 Sherwood Place. This site was selected based on the GIS data; however the pipe outfall could not be located in the field. The location where the outfall should have been located was heavily vegetated. However, a sump location was evident where runoff had ponded in the past; refer to photo on right. It could not be determined if the GIS data was incorrect or if the existing pipe had been blocked or plugged with sediment. The area draining to the sump location is approximately 1.0 acres and consists of residential areas. Approximately 30.2% (0.31 acres) of the drainage area is impervious.

The homeowner at this location had mentioned the adjacent property owners at 1115 Emmorton Road had recently paved an additional parking lot on their property, causing runoff to flow towards the problem area. A small yard inlet was installed; however it is not located in the sump location and does not intercept the parking lot runoff. Based on the GIS data, runoff from Sherwood Place should flow towards Emmorton Road; however it has been blocked by development. Further investigation of the ponding water and the possible effect of the adjacent development, for the benefit of the property resident, is recommended at this location. At this point, it can not be determined whether this site is a good candidate for a BMP facility, though the impervious area is small relative to other available sites.

4.1.8 Existing Stormwater Management Sites

Three existing stormwater management facilities were identified within the watershed and investigated as possible retrofit sites. A fourth privately owned site, Brook Hill Manor, was identified but excluded from the investigation since it is located entirely within the City of Bel Air.

Two of the existing facilities are infiltration devices along East Ring Factory Road. As previously discussed, the devices are located at Site 1 and Site 2 and detailed on the Gracelyn Section 1 As-Built Plans, dated April 1990. Both devices are not working properly, appear clogged with sediment and are ponding water. Refer to Section 4.4 Water Quality Improvement Summary for retrofit recommendations.

The last facility investigated is located at Christ Our King Presbyterian Church on the corner of Lexington Road and Emmorton Road. According to the As-Built plans, dated August 1988, an infiltration trench was installed within a detention basin area (see photo, below left) to treat runoff from the parking lot and proposed building addition. The drainage area to the facility is 2.45 acres and consists of the parking lot, a portion of the buildings and adjacent lawn. Approximately 42.0% (1.03 acres) of the drainage area is impervious

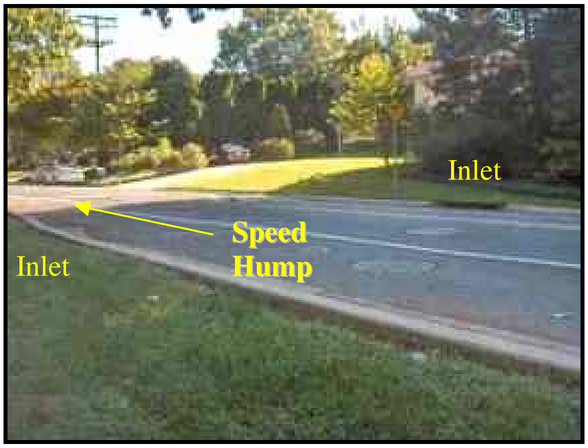
The facility appears well maintained with minimal standing water in the cleanout pipe. The stormwater inlet (below, right) does have some sediment accumulation, but appears to be infiltrating properly. A discussion with an employee of the church confirmed that the facility is maintained and does function properly in all but “historic level” (Tropical Storm Isabel was noted as one such event) rainfall events. Based on precipitation data (www.weatherunderground.com), the Bel Air area received approximately 0.05” of rain two days before our September 17th investigation; a total of 0.35” in the week preceding.

Although this was considered a possible location for a BMP retrofit based on the criteria, the facility is in good condition, appears to be continually maintained and is working properly. The cost of retrofitting this facility would not be justified by the minimal additional water quality benefits.

4.2 Additional Drainage Concerns

Two locations on East Ring Factory Road were identified by private property owners as areas where runoff ponds along the roadway, causing drainage and safety concerns.

The first location is at 208 East Ring Factory Road. Two combination inlets are located immediately upstream of a speed hump (photo below, left). It appears that the gutter pan has cracked and settled around both inlets, causing runoff to pond adjacent to the grates in the low areas (photo below, right). Based on the field investigation, several additional localized low points are located along the gutter line on both sides of the roadway.



The inlets appear in good condition and are clean of debris and sediment. The inlets convey runoff into a culvert under East Ring Factory Road, which discharges at Site 3.

The second location is at 102 East Ring Factory Road. Two inlets are located immediately downstream of a speed hump. Roadway runoff is conveyed along the gutter pan into the inlets. However, it appears that the center of the roadway has settled upstream of the speed hump (photo below, left) causing runoff to pond in this location rather than flow towards one of the inlets. Based on field observation, it appeared that the inlet on the west side of the roadway may not be located at the sump (photo below, right).

The inlets appear in good condition and are clean of debris and sediment. The inlets convey runoff into a culvert under East Ring Factory Road, which discharges at Site 6.

The failure of the pavement and pavement subgrade in localized areas appears to be the cause of the ponding issues outlined in the two sites above. These problems are best addressed as part of a pavement improvement or resurfacing project or task. By patching the areas with failing subgrade and/or gutter pan and resurfacing, low areas can be eliminated, providing positive flow towards the existing inlets.

4.3 Water Quality Improvement Summary

Seven sites were identified and investigated to determine the feasibility of installing at stormwater management facility at that location. Criteria such as amount of impervious pavement, available land between the outfall and the tributary channel, conditions of the existing facility and possible permit issues were taken into consideration. Table 7, below, summarizes the total and impervious drainage area to each site.

Site	Drainage Area (ac)	Impervious Area			Total Impervious (acres/percentage)
		Rooftop	Driveway/Parking	Roadway	
1	18.97	1.97	1.43	5.65	9.05 / 47.7%
2	19.74	1.97	1.11	2.09	5.16 / 26.1%
3	8.87	0.57	0.29	0.53	1.40 / 15.7%
4	2.92	0.18	0.20	0.57	0.94 / 32.2%
5	24.47	2.67	1.48	2.60	6.75 / 27.6%
6	43.98	3.64	2.59	3.82	10.04 / 22.8%
7	1.02	0.09	0.08	0.14	0.31 / 30.2%

Table 7: BMP Hydrologic Site Summary

Although most of the sites have a high percentage of impervious area, the majority of the sites were not considered good candidates for BMP installation based on other criteria. Site 5 and Site 7 were not considered for stormwater management facility installation since the storm drain outfall pipes could not be located. Site 3 was not recommended due to the anticipated environmental permit issues and Site 1 since the existing outfall channel is vegetated and stable, already providing some water quality treatment. Although Site 6 may have permit issues as well, the outfall channel does not currently provide any water quality treatment.

Site 2 and Site 4 are recommended for consideration of BMP retrofit and BMP installation or outfall stabilization improvement, respectively. Each site currently conveys sediment directly into the tributary through erosion within the outfall channel or at the confluence with the tributary. Improvements such as removing the riprap and/or concrete within the channels and replacing the linings with vegetation would provide increased water quality treatment. Retrofitting a stormwater quality BMP at these locations will require consideration of the benefits versus the impacts to surrounding areas. As it may not be feasible to size a BMP to provide full treatment (based on current MDE guidelines) of all impervious areas to those points, incremental treatment of a portion of the total water quality volume should be considered. This will still provide significant water quality benefit for typical runoff events. Further discussion of these water quality goals and the impact benefit analysis is recommended for the next stage of design.

4.4 Additional Data Required

In order to develop more comprehensive recommendations for Water Quality BMP Retrofit improvements, additional information including detailed survey, wetland delineation and utility designation of proposed BMP locations and potential construction access areas is required. Summary maps of the areas needed will be prepared upon determination of the sites chosen for additional design.

5.0 REFERENCES

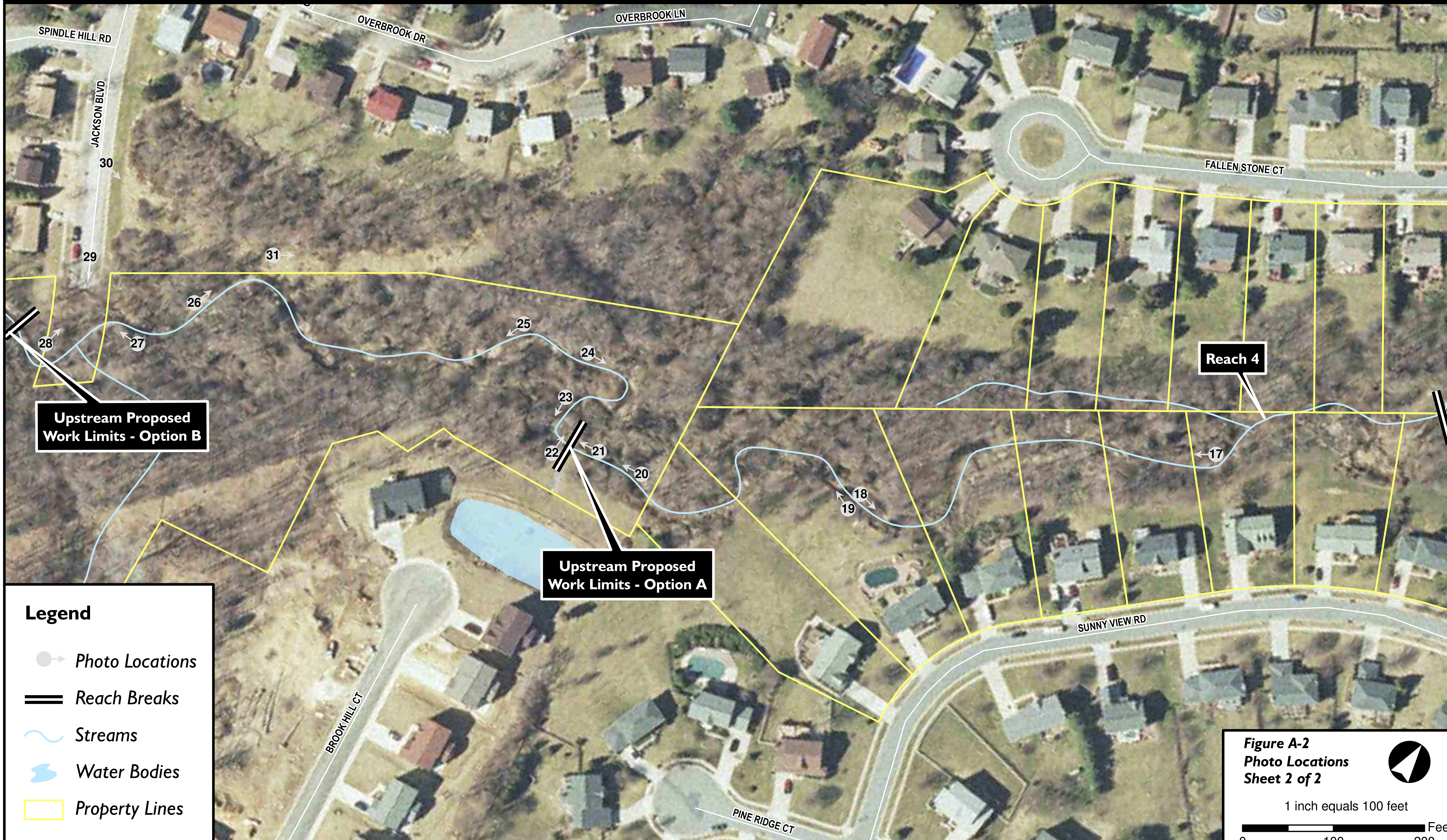
1. United States Department of Agriculture (USDA). 1985. *List of Hydric Soils of the State of Maryland*. Soil Conservation Service, Washington, D.C.
2. USDA. 1976. *Soil Survey of Harford County, Maryland*. Soil Conservation Service, Washington, D.C.
3. Cramer, M., K. Bates, D. Miller, K. Boyd, L. Fotherby, P. Skidmore, and T. Hoitsma. 2003. "Integrated Streambank Protection Guidelines", Co-published by the Washington Departments of Fish & Wildlife, Ecology, and Transportation. Olympia, Washington, 435 pp.
4. Maryland Hydrology Panel, "Application of Hydrologic Methods in Maryland", January 2005, revised August 2006.
5. Mecklenburg, D. and Ohio Department of Natural Resources. 2006. The Reference Reach Spreadsheet, v.4.3L.
6. Schumm, S.A. 1969. River metamorphosis. Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, HY 1, p. 255-273.
7. U.S. Department of Agriculture, Soil Conservation Service, "Urban Hydrology for Small Watersheds", Technical Release Number 55, June 1986.
8. U.S. Department of Agriculture, Soil Conservation Service, "TR-20 Computer Program for Project Formulation", Technical Release Number 20, February 1992.

APPENDIX A

MEADOWRIDGE CT
CRABTREE SQ
ASHLEY WAY
JACKSON BLVD
DEWBERRY WAY
THREE ACRE LN
FALLEN STONE CT
PEQUOT DR
OVERBROOK LN
OVERBROOK CT
SOUTHWORTH CT
OVERBROOK DR
SUNNY VIEW RD
MILLSTONE CT
MACPHAIL RD
FEATHERSTONE CT
PINE RIDGE CT
SPINDLE HILL RD
BROOK HILL CT
RING FACTORY RD
RASBERRY HILL CT
Reach 1
Reach 2
Reach 3
Reach 4
Downstream Proposed Work Limits
Upstream Proposed Work Limits - Option B
Upstream Proposed Work Limits - Option A
Legend
Reach Breaks
Manhole
Streams
Water Bodies
Contours
Property Lines
Sewerlines
Buildings
Figure A-1
Project Overview
1 inch equals 200 feet
Feet
0
200
400

Sunnyview Stream Restoration Site

Sunnyview Stream Restoration Site



APPENDIX B


Photo 1 – View of Bynum Run facing downstream from tributary confluence


Photo 2 – View of Bynum Run facing upstream from tributary confluence

Photo 3 – Facing downstream along tributary toward Bynum Run confluence

Photo 4 – Facing upstream toward the riffle at the downstream proposed work limits

Photo 5 – View of bank erosion along the upstream right bank located just upstream of the downstream proposed work limits

Photo 6 – View of concrete placed along the upstream right bank located just upstream of the downstream proposed work limits

Photo 7 – View of proposed construction access from Macphail Road

Photo 8 – Representative view (facing upstream) of an over-widened section within a wooded riparian zone


Photo 9 – Representative view (facing downstream) within a wooded riparian zone — note the sediment deposition


Photo 10 – View facing downstream at the Reach 1 and 2 interface

Photo 11 – View facing upstream from the Reach 1 and 2 interface

Photo 12 – Downstream view of bank erosion along a tight meander bend

Photo 13 – View of proposed construction access from Ring Factory Road

Photo 14 – View of proposed construction access from Ring Factory Road


Photo 15 – Upstream view toward the sediment transport sampling buckets


Photo 16 – Representative view (facing downstream) near the Reach 3 and 4 interface


Photo 17 – Upstream view of bank erosion—note exposed roots


Photo 18 – Downstream view of bank erosion along a residential yard


Photo 19 – Upstream view of existing scour pool—to remain following restoration


Photo 20 – Facing upstream toward the riffle at Option A of the upstream proposed work limits

Photo 21 – Facing upstream toward bank erosion located just upstream of Option A of the upstream proposed work limits

Photo 22 – Upstream view of a riffle located directly upstream of bank erosion shown in Photos 21 and 23

Photo 23 – Facing downstream toward bank erosion located just upstream of Option A of the upstream proposed work limits

Photo 24 – Downstream view of a tight meander bend with toe scour and potential for development of a chute cut-off channel

Photo 25 – Upstream view of a riffle feature located just upstream of the tight meander bend

Photo 26 – Downstream view of bank erosion where established woody vegetation along the top-of-bank line is absent

Photo 27 – Upstream view of an eight-foot high eroding streambank located directly adjacent to Jackson Boulevard

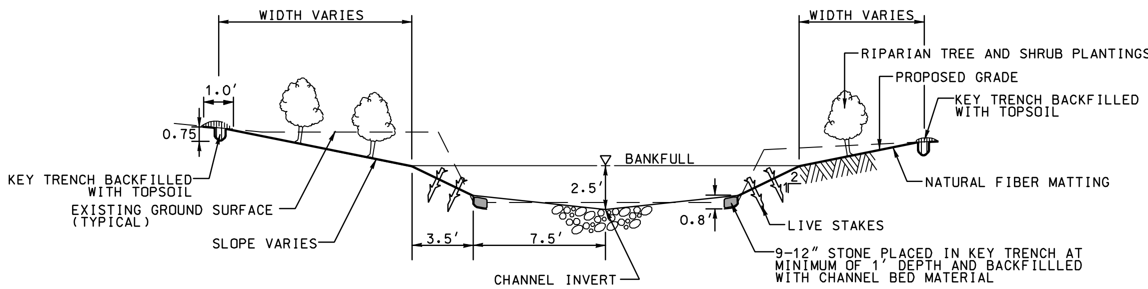
Photo 28 – Downstream view of an eight-foot high eroding streambank located directly adjacent to Jackson Boulevard

Photo 29 – View of the guardrail at the end of Jackson Boulevard located directly adjacent to the channel

Photo 30 – View of the proposed construction access from Jackson Boulevard

Photo 31 – View of the proposed construction access from Jackson Boulevard

APPENDIX C



TYPICAL SECTION 1- RIFFLE ALONG EXISTING CHANNEL ALIGNMENT

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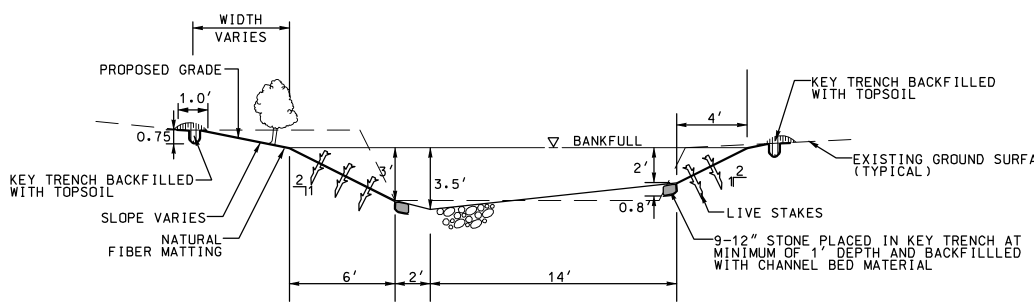

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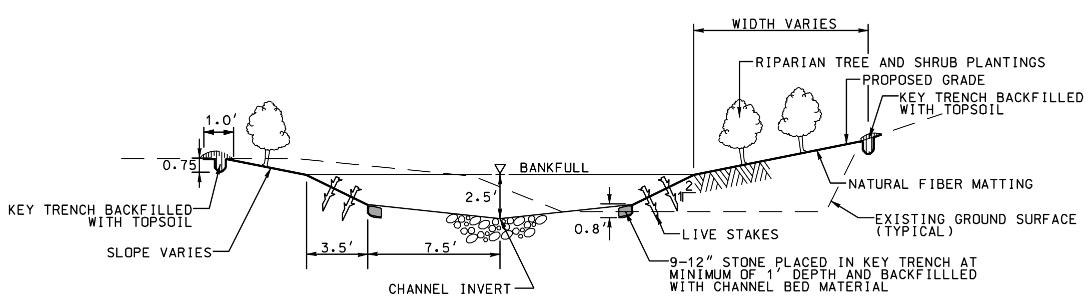

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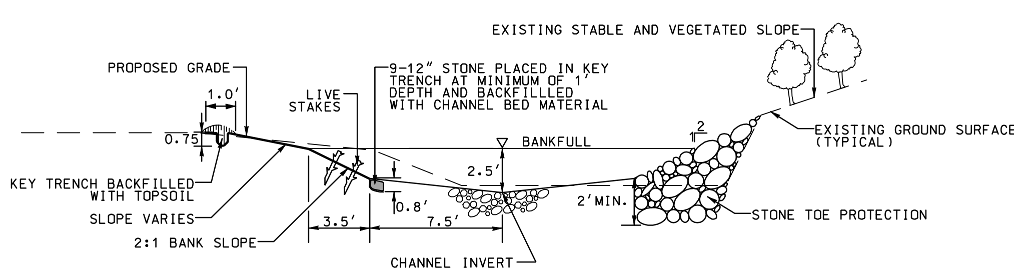

TYPICAL SECTION 4- RIFFLE ALONG SLIGHT CHANNEL SHIFT WITH STONE TOE PROTECTION

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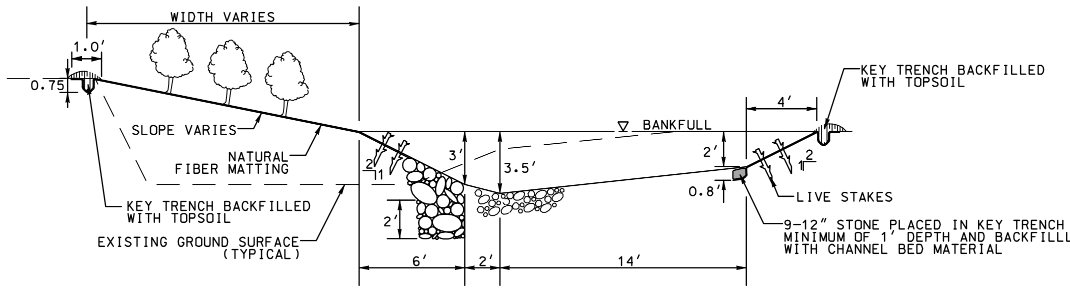

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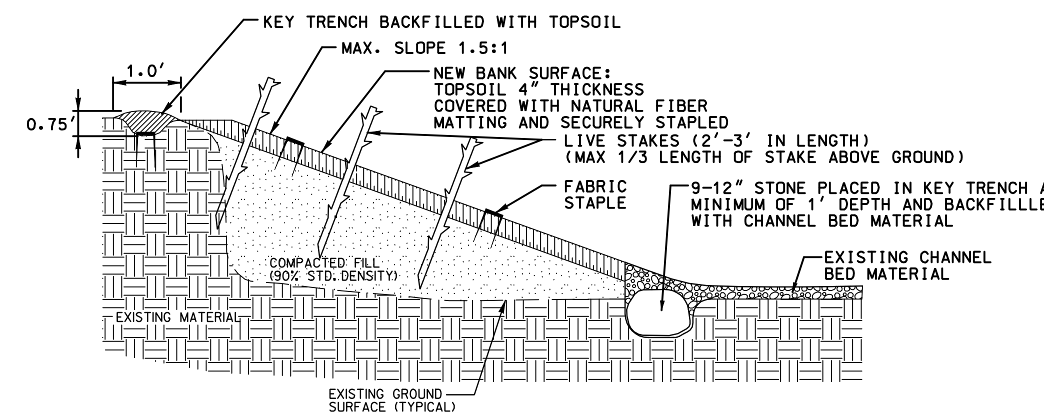

STREAMBREAK IN FILL WITH STONE KEY

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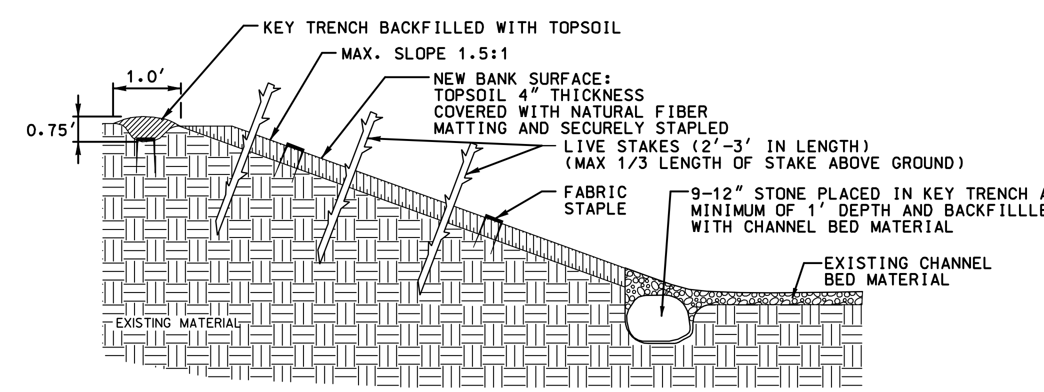

STREAMBREAK IN CUT WITH STONE KEY

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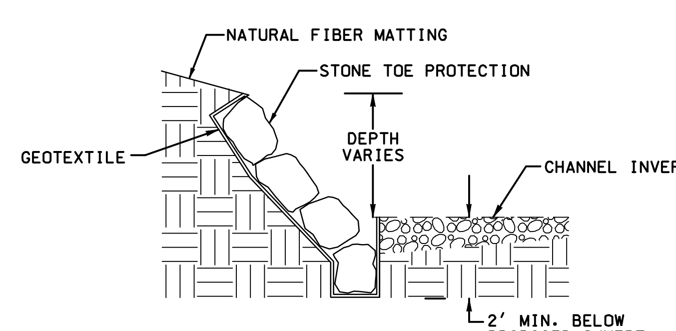

STONE TOE PROTECTION

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REVISIONS	SUNNYVIEW STREAM RESTORATION AND WATER QUALITY IMPROVMENTS			
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	DRAWN: AS			
	CHECKED: CJB		ROAD NO.	JOB NO.
	SCALE:	DWG. OF	FILE NO.	

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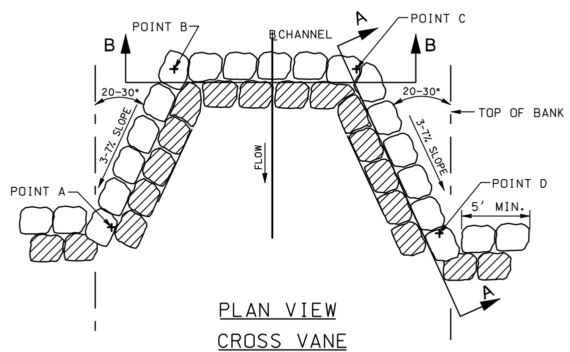

PLAN VIEW
CROSS VANE
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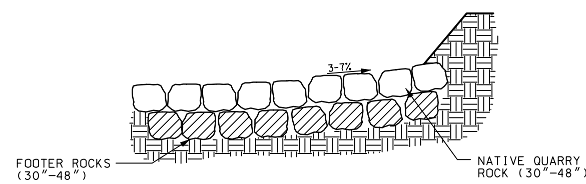

PROFILE VIEW 'A-A'
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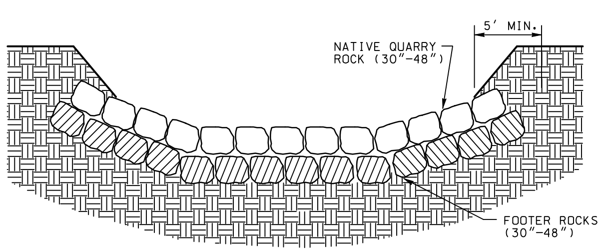

CROSS SECTION 'B-B'
CROSS VANE
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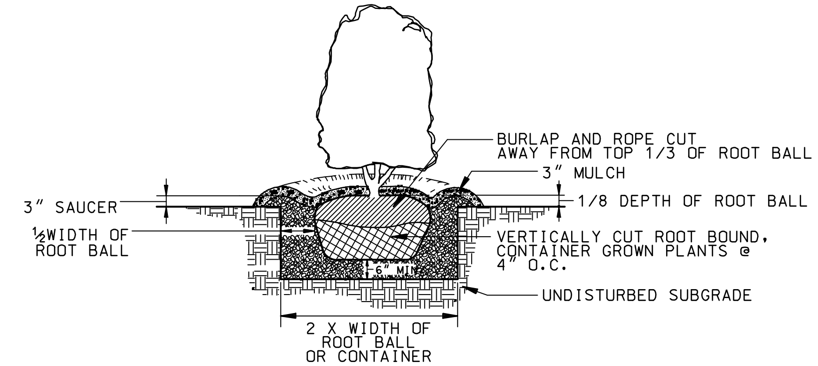
*** NOTE: ALL ROCKS OR STONES ARE NATURAL STONE, BOULDERS.



SHRUB PLANTING DETAIL
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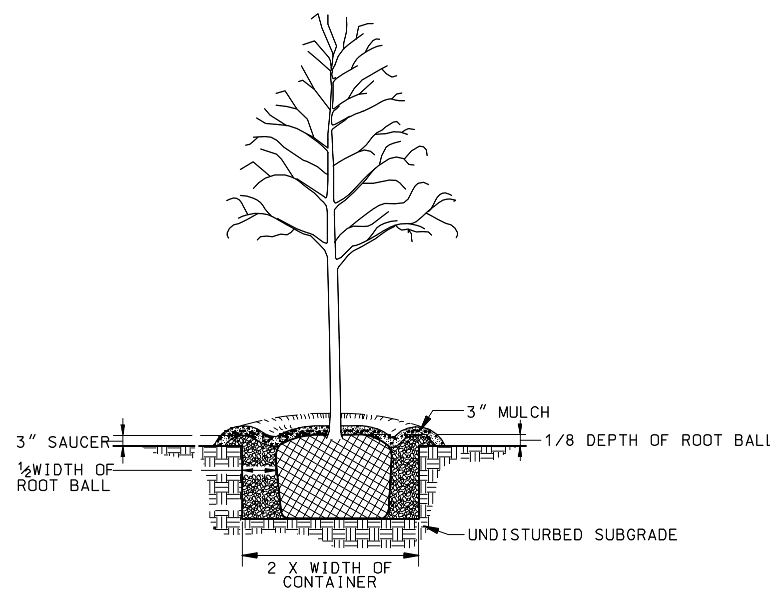

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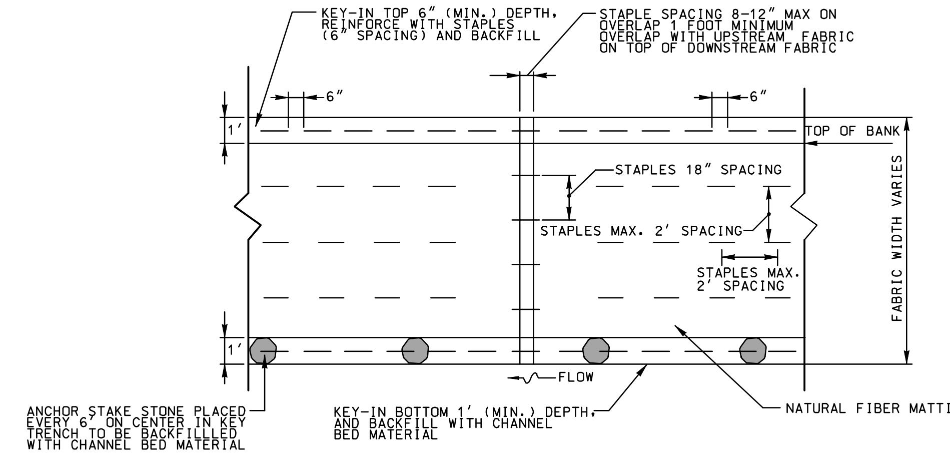

NATURAL FIBER MATTING WITH STONE KEY
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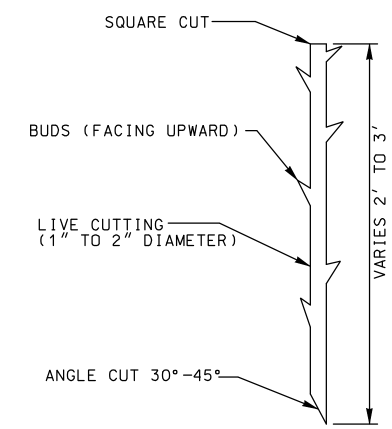

LIVE STAKE
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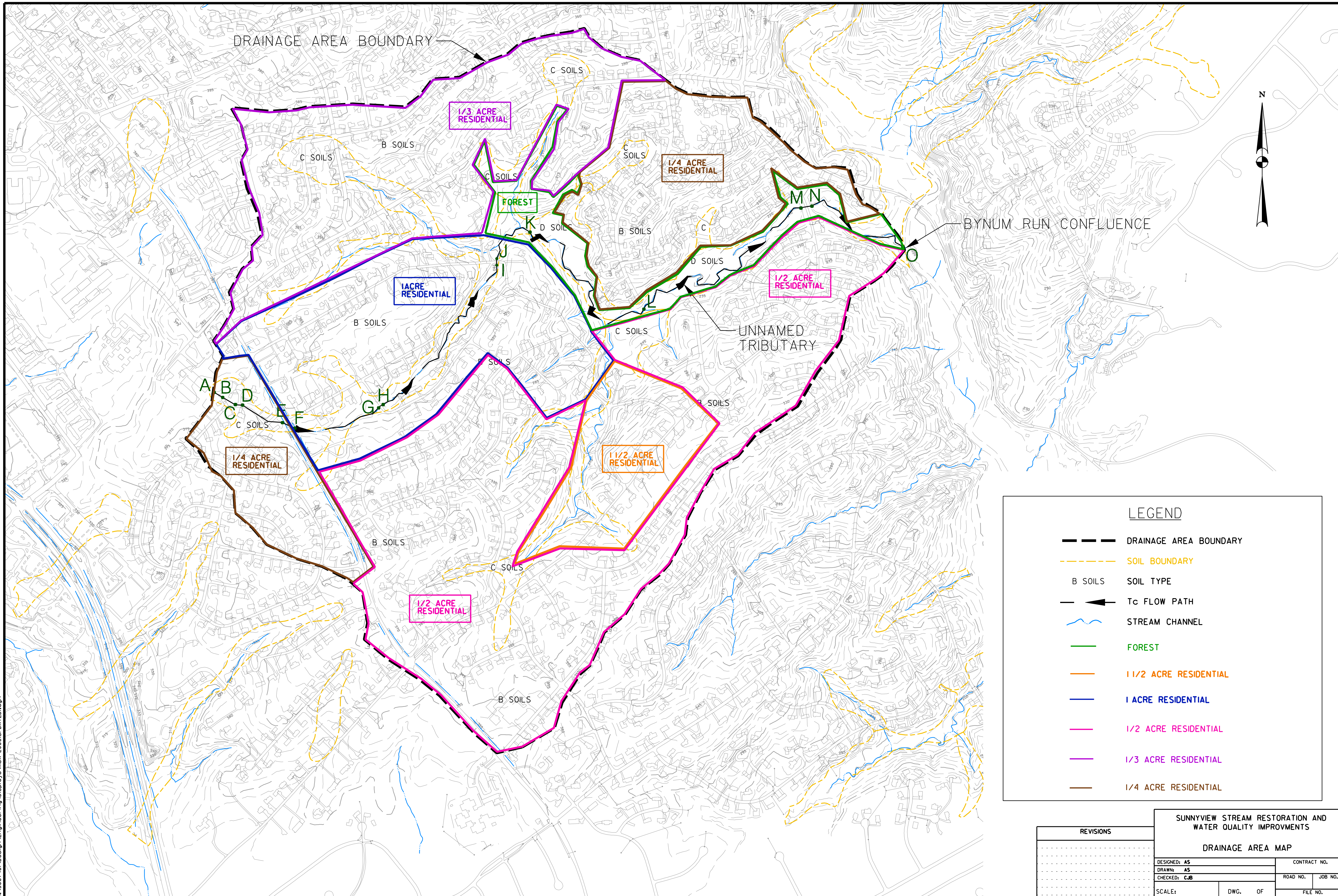
	SCALE: DWG. OF	FILE NO.	

APPENDIX D



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APPENDIX E

TR-55 TIME OF CONCENTRATION WORKSHEET

DRAINAGE AREA: Sunnyview Stream Restoration

BY: ADS
DATE: 6/20/2007

OVERLAND FLOW

Flow Segment Name	AB		
Surface Description	unpaved	(See Table 3-1)	
Roughness Coefficient	0.24		
Land Slope (ft/ft)	0.037		
Flow Length (ft) [100' max]	100.00		
Two-Year Rainfall (in.)	3.20		
Flow Time (hr.)	0.1862		0.186

SHALLOW CONCENTRATED FLOW

Flow Segment Name	BC	CD			
Flow Length (ft)	169.45	67.6677			
Paved or Unpaved	unpaved	paved			
Land Slope (ft/ft)	0.024	0.037			
Flow Velocity (ft/sec.)	2.5068	3.9134			
Flow Time (hr.)	0.0188	0.0048			0.024

CHANNEL FLOW

Flow Segment Name	DE	FG	HI	JK	KL	LM	NO	
Flow Depth (ft)	1	1	1	1	1	1	1	
Bottom Width (ft)	2	2	2	2	2	2	2	
Side Slope (Z1)	2	2	2	2	2	2	2	
Side Slope (Z2)	2	2	2	2	2	2	2	
Manning's Coefficient	0.040	0.040	0.040	0.04	0.04	0.04	0.04	
Flow Length (ft)	394.11	808.36	1793.03	533.45	1873.12	2193.99	1018.04	
Channel Slope (ft/ft)	0.024	0.040	0.036	0.0281	0.0134	0.0114	0.01473	
Flow Velocity (ft/sec.)	4.1409	5.3977	5.1310	4.5192	3.1226	2.8767	3.2714109	
Flow Time (hr.)	0.0264	0.0416	0.0971	0.0328	0.1666	0.2118536	0.0864424	0.6628

PIPE FLOW (Assuming full flow)

Flow Segment Name	EF	GH	IJ	MN	
Pipe Diameter (ft)	2.00	2.00	6.00	8.00	
Manning's Coefficient	0.013	0.013	0.013	0.013	
Pipe Slope (ft/ft)	0.024	0.017	0.036	0.011	
Pipe Length (ft)	50.00	30.00	55.00	80.00	
Flow Velocity (ft/sec.)	11.156	9.306	28.420	19.374	
Flow Time (hr.)	0.0012	0.001	0.001	0.001	0.004

TIME OF CONCENTRATION (hr.)/(min) 0.876 52.58

SCS TR-55 RUNOFF CURVE NUMBER AND TIME OF CONCENTRATION COMPUTATION SHEET

JOB NAME: Sunnyview Stream Restoration
Harford County

DATE: 11/07/07
JOB NO.: 5067-01

COMPUTED BY: ADS
CHECKED BY: CJB

STUDY POINT: DS Limits

CONDITION: ULTIMATE
X EXISTING

RUNOFF CURVE NUMBER COMPUTATION

HYDROLOGIC SOIL GROUP	LAND USE	RUNOFF CURVE NO.	Area (ft^2)	AREA (ac)	RCN x A
	Pavement	98	0	0.00	0.00
B	Residential - 1 1/2 acre lot	66	1293632	29.70	1960.05
C	Residential - 1 1/2 acre lot	78	276479	6.35	495.07
B	Residential - 1/4 acre lot	75	3818615	87.66	6574.75
C	Residential - 1/4 acre lot	83	370784	8.51	706.50
B	Residential - 1/3 acre lot	72	3668865	84.23	6064.24
C	Residential - 1/3 acre lot	81	697153	16.00	1296.36
B	Residential - 1/2 acre lot	70	8108905	186.15	13030.84
C	Residential - 1/2 acre lot	80	1196374	27.46	2197.20
B	Residential - 1 acre lot	68	3547945	81.45	5538.57
C	Residential - 1 acre lot	79	739341	16.97	1340.86
C	Forest	70	199724	4.59	320.95
D	Forest	77	1001717	23.00	1770.71
B	Forest	55	255911	5.87	323.12
		TOTAL	**25175446**	**577.95**	**41619.23**
				MI^2	**0.90304**

WEIGHTED RUNOFF CURVE NUMBER = $\dfrac{\textbf{TOT RCN x AC =}}{\textbf{TOTAL ACRES}}$ $\dfrac{\mathbf{41619.23}}{\mathbf{577.95}}$ = **72.0**

APPENDIX F

TR-20 Output

```
*****************80-80 LIST OF INPUT DATA FOR TR-20 HYDROLOGY*****************

JOB TR-20                                                   NOPLOTS
TITLE     Watershed Investigation
TITLE     Sunny View
 6 RUNOFF 1   3       1       0.9018      72.700       0.8760         1
   ENDATA
 7 INCREM 6                      0.1
 7 COMPUT 7   3     3            0.0        2.75         1.02 2  1   1
   ENDCMP 1
 7 COMPUT 7   3     3            0.0        3.33         1.02 2  1   2
   ENDCMP 1
 7 COMPUT 7   3     3            0.0        4.29         1.02 2  1   5
   ENDCMP 1
 7 COMPUT 7   3     3            0.0        5.12         1.02 2  1   10
   ENDCMP 1
 7 COMPUT 7   3     3            0.0        6.41         1.02 2  1   25
   ENDCMP 1
 7 COMPUT 7   3     3            0.0        7.55         1.02 2  1   50
   ENDCMP 1
 7 COMPUT 7   3     3            0.0        8.84         1.02 2  1   99
   ENDCMP 1
   ENDJOB 2

*******************************END OF 80-80 LIST*******************************
```

```
 EXECUTIVE CONTROL INCREM    MAIN TIME INCREMENT =   .100 HOURS


 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  2.75       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. = 1            RAIN TABLE NO. = 2


 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   1


 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  3.33       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. = 2            RAIN TABLE NO. = 2


 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   2


 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  4.29       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. = 5            RAIN TABLE NO. = 2


 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   3


 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  5.12       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. =10            RAIN TABLE NO. = 2


 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   4
```



```
 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  6.41       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. =25            RAIN TABLE NO. = 2
```

```
 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   5


 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  7.55       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. =50            RAIN TABLE NO. = 2


 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   6


 EXECUTIVE CONTROL COMPUT    FROM XSECTION   3  TO XSECTION   3
    STARTING TIME =   .00       RAIN DEPTH =  8.84       RAIN DURATION =  1.00
    ANT. RUNOFF COND. = 2       MAIN TIME INCREMENT =   .100 HOURS
    ALTERNATE NO. = 1           STORM NO. =99            RAIN TABLE NO. = 2


 EXECUTIVE CONTROL ENDCMP    COMPUTATIONS COMPLETED FOR PASS   7
1
 TR20 -------------------------------------------------------------------- SCS -
                            Watershed Investigation                      VERSION
 06/21/**                          Sunny View                           2.04TEST
 10:33:41                      SUMMARY, JOB NO.   1                     PAGE   3


                             SUMMARY TABLE 1
                             ---------------
    SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL IN ORDER PERFORMED.
    A CHARACTER FOLLOWING THE PEAK DISCHARGE TIME AND RATE (CFS) INDICATES:
 F-FLAT TOP HYDROGRAPH   T-TRUNCATED HYDROGRAPH   R-RISING TRUNCATED HYDROGRAPH

 XSECTION/  STANDARD                                    PEAK DISCHARGE
 STRUCTURE   CONTROL    DRAINAGE   RUNOFF   ------------------------------------
    ID      OPERATION     AREA     AMOUNT   ELEVATION   TIME     RATE    RATE
                        (SQ MI)     (IN)      (FT)      (HR)     (CFS)   (CSM)

 RAINFALL OF   2.75 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.
 RAINTABLE NUMBER  2,    ARC 2
 MAIN TIME INCREMENT   .100 HOURS

    ALTERNATE    1   STORM    1
    ---------------------------
 XSECTION   3  RUNOFF      .90      .69        ---     12.48      200    222.2

 RAINFALL OF   3.33 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

    ALTERNATE    1   STORM    2
    ---------------------------
 XSECTION   3  RUNOFF      .90     1.05        ---     12.46      324    360.0

 RAINFALL OF   4.29 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

    ALTERNATE    1   STORM    5
    ---------------------------
 XSECTION   3  RUNOFF      .90     1.71        ---     12.44      563    625.6

 RAINFALL OF   5.12 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.
```

```
   ALTERNATE    1   STORM   10
   ---------------------------
XSECTION   3  RUNOFF      .90     2.35        ---     12.43      780    866.7

RAINFALL OF   6.41 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

   ALTERNATE    1   STORM   25
   ---------------------------
XSECTION   3  RUNOFF      .90     3.40        ---     12.42     1156   1284.4
1
TR20 -------------------------------------------------------------------- SCS -
                           Watershed Investigation                     VERSION
06/21/**                          Sunny View                          2.04TEST
10:33:41                      SUMMARY, JOB NO.   1                    PAGE   4


                            SUMMARY TABLE 1
                            ---------------
   SELECTED RESULTS OF STANDARD AND EXECUTIVE CONTROL IN ORDER PERFORMED.
   A CHARACTER FOLLOWING THE PEAK DISCHARGE TIME AND RATE (CFS) INDICATES:
F-FLAT TOP HYDROGRAPH   T-TRUNCATED HYDROGRAPH   R-RISING TRUNCATED HYDROGRAPH

XSECTION/  STANDARD                                    PEAK DISCHARGE
STRUCTURE   CONTROL    DRAINAGE   RUNOFF   ------------------------------------
   ID      OPERATION     AREA     AMOUNT   ELEVATION   TIME      RATE    RATE
                       (SQ MI)     (IN)      (FT)      (HR)     (CFS)   (CSM)


RAINFALL OF   7.55 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

   ALTERNATE    1   STORM   50
   ---------------------------
XSECTION   3  RUNOFF      .90     4.37        ---     12.42     1483   1647.8

RAINFALL OF   8.84 inches AND  24.00 hr DURATION, BEGINS AT    .0 hrs.

   ALTERNATE    1   STORM   99
   ---------------------------
XSECTION   3  RUNOFF      .90     5.52        ---     12.41     1881   2090.0
1
TR20 -------------------------------------------------------------------- SCS -
                           Watershed Investigation                     VERSION
06/21/**                          Sunny View                          2.04TEST
10:33:41                      SUMMARY, JOB NO.   1                    PAGE   5


                            SUMMARY TABLE 3
                            ---------------
     STORM DISCHARGES (CFS) AT XSECTIONS AND STRUCTURES FOR ALL ALTERNATES
     QUESTION MARK (?) AFTER: OUTFLOW PEAK - RISING TRUNCATED HYDROGRAPH.

XSECTION/          DRAINAGE
STRUCTURE            AREA      STORM NUMBERS..........
  ID               (SQ MI)          1         2         5         10         25


XSECTION    3          .90
---------------------------
   ALTERNATE    1                  200       324       563        780       1156
```

```
                                   SUMMARY TABLE 3
                                   ---------------
          STORM DISCHARGES (CFS) AT XSECTIONS AND STRUCTURES FOR ALL ALTERNATES
          QUESTION MARK (?) AFTER: OUTFLOW PEAK - RISING TRUNCATED HYDROGRAPH.

 XSECTION/          DRAINAGE
 STRUCTURE            AREA       STORM NUMBERS..........
   ID               (SQ MI)         50        99


 XSECTION    3           .90
 ---------------------------
    ALTERNATE    1                 1483      1881
1
 TR20 -------------------------------------------------------------------- SCS -
                           Watershed Investigation                       VERSION
 06/21/**                         Sunny View                            2.04TEST



 END OF  1 JOBS IN THIS RUN

                     SCS  TR-20,  VERSION 2.04TEST
                                      FILES

     INPUT  = Saw2985.dat                                , GIVEN DATA FILE
     OUTPUT = Saw2985out.out                             , DATED 06/21/**,10:33:41

              FILES GENERATED - DATED 06/21/**,10:33:41

                               NONE!


           TOTAL NUMBER OF WARNINGS =   0,  MESSAGES =   0

                     *** TR-20 RUN COMPLETED ***
```

Regression Equation Data (from GISHydro)

```
Watershed Statistics for: Sunnyview unnamed trib to Bynum Run
GISHydro Release Version Date: September 5, 2006
Hydro Extension Version Date:  September 4, 2006
Analysis Date:                 June 22, 2007

Data Selected:
     Quadrangles Used: jarrettsville, bel_air
     DEM Coverage: NED DEMs
     Land Use Coverage: 2002 MD/DE Landuse
     Soil Coverage: STATSGO Soils
     Hydrologic Condition: (see Lookup Table)
     Impose NHD stream Locations: Yes
     Outlet Easting:         458630 m. (MD Stateplane, NAD 1983)
     Outlet Northing:        207021 m. (MD Stateplane, NAD 1983)

Findings:
     Outlet Location:        Piedmont
     Outlet State:           Maryland
     Drainage Area           0.9 square miles
       -Piedmont (100.0% of area)
     Channel Slope:          119.9 feet/mile
     Land Slope:             0.055 ft/ft
     Urban Area:             94.2%
     Impervious Area:        35.3%
 ******************************************************
     URBAN DEVELOPMENT IN WATERSHED EXCEEDS 15%.
     Calculated discharges from USGS Regression
     Equations may not be appropriate.
 ******************************************************
     Time of Concentration: 1.3 hours  [W.O. Thomas, Jr. Equation]
     Time of Concentration: 1.5 hours  [From SCS Lag Equation * 1.67]
     Longest Flow Path:     1.67 miles
     Basin Relief:          123.1 feet
     Average CN:            74
     % Forest Cover:        4.0
     % Storage:             0.0
     % Limestone:           0.0
     Selected Soils Data Statistics:
        % A Soils:          5.9
        % B Soils:          76.1
        % C Soils:          11.7
        % D Soils:          4.7
     STATSGO Soils Data Statistics (used in Regression Equations):
        % A Soils:          5.9
        % B Soils:          76.2
        % C Soils:          11.7
        % D Soils:          4.7
     2-Year,24-hour Prec.:  3.32 inches
     Mean Annual Prec.:     48.91 inches
```

Fixed Region Peak Flow Estimates for: Sunnyview unnamed trib to Bynum Run
GISHydro Release Version Date: September 5, 2006
Hydro Extension Version Date: September 4, 2006
Analysis Date: June 22, 2007

Geographic Province(s):
-Piedmont (100.0% of area)

Q(1.25): 163 cfs
Q(1.50): 222 cfs
Q(1.75): 258 cfs
Q(2): 286 cfs
Q(5): 532 cfs
Q(10): 755 cfs
Q(25): 1120 cfs
Q(50): 1460 cfs
Q(100): 1870 cfs
Q(200): 2350 cfs
Q(500): 3140 cfs

Area Weighted Prediction Intervals (from Tasker)

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
1.25	123	215	109	243	80	331	69	387
1.5	173	284	155	317	118	418	103	480
1.75	203	329	183	365	140	477	122	546
2	226	362	203	402	156	523	137	597
5	438	647	402	704	324	874	291	975
10	632	903	584	977	479	1190	433	1320
25	938	1340	867	1450	712	1760	645	1940
50	1210	1760	1110	1920	902	2370	812	2630
100	1520	2300	1380	2520	1100	3180	976	3580
200	1860	2980	1680	3300	1290	4290	1130	4890
500	2390	4140	2110	4670	1560	6340	1330	7400

Individual Province Tasker Analyses Follow:

Flood frequency estimates for
Sunnyview unnamed trib to Bynum Run
REGION: Piedmont Urban
area= 0.90:impervious area = 35.30 :skew= 0.58

Return Period	Discharge (cfs)	Standard Error of Prediction (percent)	Equivalent Years of Record	Standard Error of Prediction (logs)
1.25	163.	41.7	3.17	0.1739
1.50	222.	36.9	3.58	0.1552
1.75	258.	35.7	3.93	0.1503
2.00	286.	35.1	4.32	0.1481
5.00	532.	28.5	12.63	0.1216
10.00	755.	26.2	22.97	0.1119
25.00	1120.	26.0	36.33	0.1110
50.00	1460.	27.7	41.89	0.1182
100.00	1870.	30.8	43.08	0.1306
200.00	2350.	34.8	41.47	0.1469
500.00	3140.	41.3	37.95	0.1722

P R E D I C T I O N   I N T E R V A L S

Return	50 PERCENT		67 PERCENT		90 PERCENT		95 PERCENT	
Period	lower	upper	lower	upper	lower	upper	lower	upper
1.25	123.	215.	109.	243.	80.	331.	69.	387.
1.50	173.	284.	155.	317.	118.	418.	103.	480.
1.75	203.	329.	183.	365.	140.	477.	122.	546.
2.00	226.	362.	203.	402.	156.	523.	137.	597.
5.00	438.	647.	402.	704.	324.	874.	291.	975.
10.00	632.	903.	584.	977.	479.	1190.	433.	1320.
25.00	938.	1340.	867.	1450.	712.	1760.	645.	1940.
50.00	1210.	1760.	1110.	1920.	902.	2370.	812.	2630.
100.00	1520.	2300.	1380.	2520.	1100.	3180.	976.	3580.
200.00	1860.	2980.	1680.	3300.	1290.	4290.	1130.	4890.
500.00	2390.	4140.	2110.	4670.	1560.	6340.	1330.	7400.

Warning: VPI is negative 11